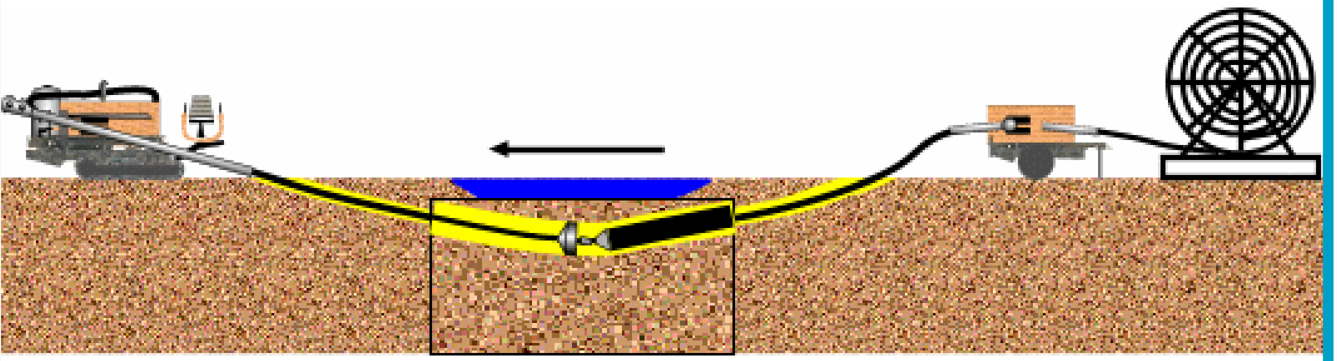
TECHNICAL NOTE: HORIZONTAL DIRECTIONAL DRILLING (Guided Boring) with PLEXCO Pipe



Rev. 3/00 Trenchless Technology Bulletin No. 1

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DIRECTIONAL DRILLING (Guided Boring) WITH POLYETHYLENE PIPE

INTRODUCTION

Horizontal directional drilling (HDD) is a method for installing subsurface piping and conduit by using a surface-mounted rig, first to drill a guided hole along a bore path consisting of a shallow arc and then to pull a string of pipe into the hole. Pullback is facilitated by a back-reamer, which enlarges the hole to approximately one and a half times the pipe's diameter. Drilling fluids are normally injected into the borehole to stabilize the hole and lubricate the pipe and drill-string. Tracking equipment is used to guide and direct the drilling.

Horizontal directional drilling of service lines, mains and conduits in urban and dense suburban areas greatly reduces costs of pipe installation, particularly social costs by minimizing traffic disruption, surface restoration, and preserving landscaping. HDD is also cost effective for pipeline crossings under major thoroughfares, railroads, rivers, and other waterways. The minimal surface impact also makes HDD useful in environmentally sensitive areas. The success of HDD is evident by the increase in sales of directional drilling units. From the mid-eighties to the mid-nineties sales of drilling units increased by 100-fold.

Polyethylene (PE) pipe is one of the most commonly used pipes for directional drilling primarily because of polyethylene's abrasion resistance, flexibility, toughness, and butt-fused joints which enables lengths of pipe to be joined together to form an essentially continuous pipeline. During pullback, butt-fused joints have a tensile strength as strong as the pipe itself. Polyethylene pipe is also available in coils. Both high-density and medium-density polyethylene pipes may be installed by HDD. In preparing for an HDD installation, the designer must determine the route for the pipe (called a bore path), estimate the load applied to the pipe during pullback, and select an appropriate pipe diameter and DR. Many of the design equations suitable for pipes installed in trenches are not applicable to directional drilled pipes. Soil/pipe interaction must be looked at in a new way. This Technical Note gives the designer an overview of things to consider for directional drilling PE pipes. It is by no means a comprehensive discussion of HDD. Many of the equations given in this Technical Note are approximations. Other equations and approaches are available and may be more appropriate. In addition, there are other documents that the designer should review and study prior to a directional drilled project. But, more importantly the designer should rely on people with HDD experience, ie. consultants, contractors, drillers, and other engineers to ultimately make design decisions as each application is unique and site specific.

Reference Documents

In 1995, the North American Society for Trenchless Technology (NASTT) published the Mini-Horizontal Directional Drilling Manual [1]. This document along with the "Guidelines for a Successful Directional Crossing Bid Package" [2] published by the Directional Crossing Contractors Association (DCCA) give useful and practical information for the contractor/installer. Design and technical requirements for HDD are discussed in Chapter 11, "Polyethylene Pipe for Horizontal Directional Drilling" of the Plastics Pipe Institute's Handbook of Polyethylene Piping [3]. Papers on directional drilling have been presented at NASTT No-Dig Conferences, Underground Construction Technology (UCT) conferences, and various ASCE speciality conferences on trenchless technology. Presently there is work underway in ASTM to produce a standard for HDD installation of PE pipe. The Gas Research Institute has published a generic specification in *Guidelines for the Application of Guided Horizontal Drilling to Install Gas Distribution Pipe* [4]. Additional references are listed at the end of this Technical Note.

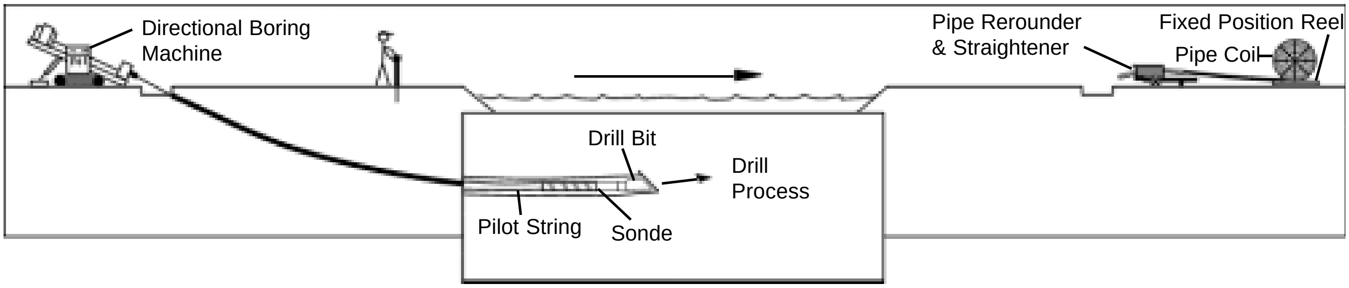
Drilling and Pullback Overview

Figure 1 is a graphic illustration of the drilling and pullback process. The drill rig is set up at the “entry pit”. A drill bit is mounted onto a drill rod. The rod is rotated and pushed into the ground by hydraulic cylinders in the drill rig. After the rod is inserted in the ground, another rod is connected to it to form a “string”. The process continues until the pilot hole has been bored under the crossing obstacle (road, waterway, pipe corridor, etc.) where it emerges in the “exit pit.” In addition to mechanical cutting due to rotation of the bit, fluid jets are sometimes used. (In rock and some special cases, the drill bit may be driven by a “mud motor”.) The direction of bore is controlled by the orientation of the drill bit which has a tapered head. Rotating the drill rod orients the taper. When the drill string is pushed without rotation, the bit will steer in the direction of the taper. On the other hand, when the drill bit is pushed with rotation, the drill string continues along the existing path. An electronic sonde is located in the drill rod just behind the drill bit. A surface receiver deciphers the sonde’s data to locate the depth, pitch, and position of the drill bit so that the driller can adjust the direction of the bit and thus control the bore path. More sophisticated electronic tracking and guidance systems which can transmit information by wire to the driller are available. For the bore, bentonite, polymer, or a combination of bentonite and polymer are mixed with water to make a drilling slurry which is injected into the hole. The drilling fluid stabilizes the hole, removes cuttings, reduces drilling torque, lubricates the drill pipe, and cools the drill bit. More importantly, the fluid will lubricate the service pipe during pullback thus reducing the pullback forces. When the fluid is mixed with the cutting it is referred to as the drilling mud. Excess mud and cuttings are removed from the entry and exit pits for disposal. Drilling muds are thixotropic and will thicken when undisturbed but, unless cementitious agents are added, their final viscosity is less than that of soft clay so they offer little “side” support to the service pipe, once it is installed.

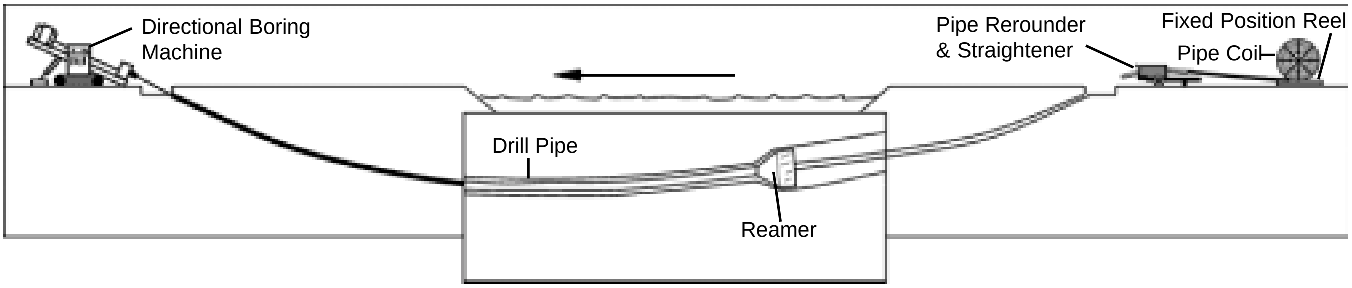
When the drill string emerges in the exit pit, the drill bit is removed and a reamer is placed on the end of the drill string. The reamer is pulled back through the bore hole to enlarge it to 120% to 150% of the service pipe diameter. (Some drillers ream to 12" larger than the pipe OD for pipes greater than 24" in diameter.) This is done to facilitate insertion of the service pipe. Since the hole is larger in diameter than the service pipe, the service pipe will be subjected to soil loads without side-support from the surrounding soil. This is a major difference between HDD pipe and pipe installed in a trench.

Figure 1: Pilot Hole Drilling, Back Reaming and Pullback

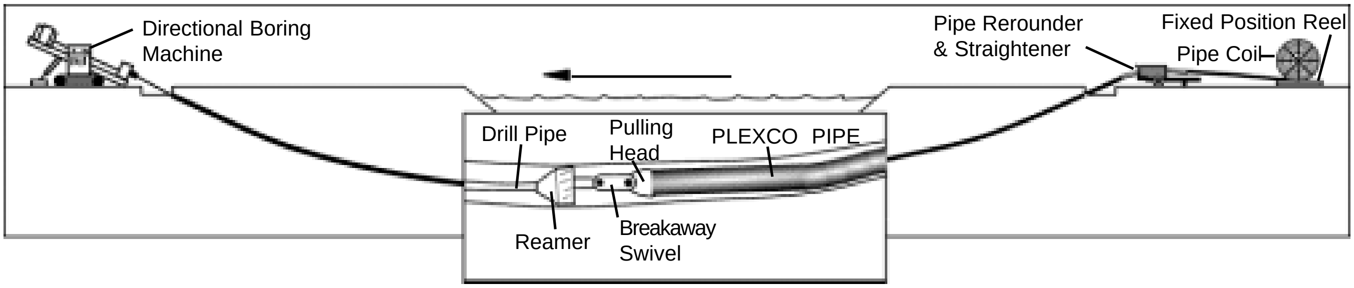
PILOT HOLE DRILLING



PREREAM



BACK REAMING & PULL BACK



In some cases, the reamer is pulled through the hole to pre-ream the hole and increasingly larger reamers are used until the hole has been bored to the desired size. After reaming is complete and in some cases during the last reaming operation, the service pipe is attached to the reamer. The service pipe is normally fused and placed on the exit pit side of the bore. (In sizes up through 6", coiled pipes can be used.) A pulling head is attached to the service pipe and connected to the reamer through a swivel, which permits the reamer to rotate without rotating the service pipe. The entire pipeline length will be pulled back continuously and in one segment through the drilling mud along the reamed bore path. Proper pipe handling, cradling, bending minimization, surface inspection, and fusion welding procedures need to be followed.

Pullback normally applies a large tensile force to the pipe. The designer and the installer must estimate this force so that the proper pipe DR is selected. A break-a-way unit, shear pin, or weak link should be placed between the pipe and reamer to protect the pipe from over stressing. ASTM F-1804 shows how to calculate the maximum pulling force permitted on PE pipes. The installer must also estimate the pulling force so that the drill rig can be sized to have sufficient pullback capacity and drill head torque. The drill operator should be trained to recognize the difference between the hydraulic pressure readings on the rig's gauges and the actual pullback force applied to the reamer and service pipe. The contractor should maintain log sheets for each bore to record at intervals equipment operating parameters such as: drilling fluid pressure, flow rate, drill thrust pressure, pull-back pressure, drill head torque, mud logs (weight of mud in and out, etc), alignment, (drill head location plots, typically at 10 ft intervals) etc.

One of the most important safety issues with directional drilling is protecting the workers and other people in the area against electrical strikes. Please contact NASTT and DCCA for more information. The NASTT Mini-Horizontal Directional Drilling Manual has a section on electrical strikes.

The HDD Industry categorizes directional drilling operations as “guided boring”, “maxi-HDD”, or “midi-HDD”. **Guided boring** or “**mini-HDD**” generally refers to bores by rigs which have less than 12,000 lbf of thrust (pull back) capacity. These rigs can bore several hundred feet in length for installing pipes up to 14" in diameter at depths typically less than 25 ft, although these limits seem to increase with each improvement in technology. Guided boring is used to place distribution lines (including service laterals) and conduits beneath streets, private property and along right-of-ways. **Maxi-HDD** generally refers to boring by rigs with thrust capacities ranging from 100,000 lbf to 750,000 lbf which are used to bore up to several thousand feet in length for placing pipes up to 54" in diameter. This includes placement of pipes under large rivers. Jobs that fall in between these two categories are sometimes called **midi-HDD**.

DR SELECTION CRITERIA

Polyethylene pipes are classified by their “Dimension Ratio (DR)”. The DR for IPS and DIPS pipe is equal to the average outside diameter divided by the minimum wall thickness. The designer is responsible for selecting the proper pipe DR for the intended application. This Technical Note is a guide that will point out to the designer many of the major considerations for DR selection and installation of PE pipes, but it may not cover every contingency encountered and therefore may not be complete. This Note should not be taken as a substitute for engineering judgment. The field of directional drilling is still under development. More appropriate equations will likely be developed with time. (Currently, university research is underway in this area.) **The designer/installer must exercise considerable engineering judgment in the use of the equations given in this document.**

The designer must select a pipe that will safely handle both (1) installation loads, which are generally of short-term duration, and (2) service loads which include earth, live and groundwater loads and which occur over the service life of the pipe. Installation pullback loads cause axial (longitudinal) tensile stresses while earth, live and groundwater loads generally cause ring (hoop) compressive stresses. Even though the ring stresses are typically much lower than the axial tensile stresses, they are just as critical to proper design as they are applied over a long period of time. Service loads also include those loads related to the fluid carrying function of the pipe such as internal pressure, surge, and vacuum. The fluid carrying requirements are addressed in PLEXCO’s Engineering Manual II and in codes and standards such as AWWA C906 and ASTM F-714. This Technical Note will focus on the forces applied to the pipe during pullback and on the earth and groundwater loads occurring after directional drilling. A detailed discussion of these topics is given in the Plastics Pipe Institute’s Handbook of Polyethylene Piping, Chapter 11, “PE Pipe for Horizontal Directional Drilling”.

SERVICE LOAD CRITERIA FOR DR SELECTION

Polyethylene is a viscoelastic material and undergoes more deformation when subjected to long-term loads than when subjected to short-term loads. For PE pipes, the long-term loading is often more critical than the short-term loading. Therefore, it is customary to analyze the long-term loading, i.e. service loads, prior to analysis of the short-term (installation) loads.

The subsurface forces acting on directional drilled pipe after installation are different than the forces acting on pipe buried in a trench. Entrenched pipe is surrounded by essentially uniform embedment, often compacted. There is significant soil/structure interaction, which both reduces the loads on the pipe via arching and increases the pipe's resistance to deflection through side support. On the other hand, the directional drilled pipe is installed in a bore hole surrounded by a slurry of drilling muds and cuttings (the mud-cuttings slurry). The bore hole is subject to deformation, raveling, sloughing, and potentially collapse. These mechanisms result in the likelihood of non-uniform and variable loads on the pipe. Earth pressures on the pipe may range from virtually no load to full overburden pressure. The deformation of the borehole and subsequent load on the pipe is dependent on the insitu soil properties as well as the viscosity and compressibility of the residual mud-cuttings slurry. Drilling mud experts indicate that mud-cuttings slurry properties vary considerably depending on insitu soil properties, groundwater conditions, and drilling techniques. Often the slurry remains saturated with the consistency of very soft clay. **Until research is done to relate these parameters to actual soil stiffness values, it is conservative to assume that unlike embedment material, the residual mud-cuttings slurry is highly compressible and offers little if any side support to the pipe.** Therefore, equations suitable for entrenched pipes may not be applicable to directionally drilled pipe.

DR Selection Process

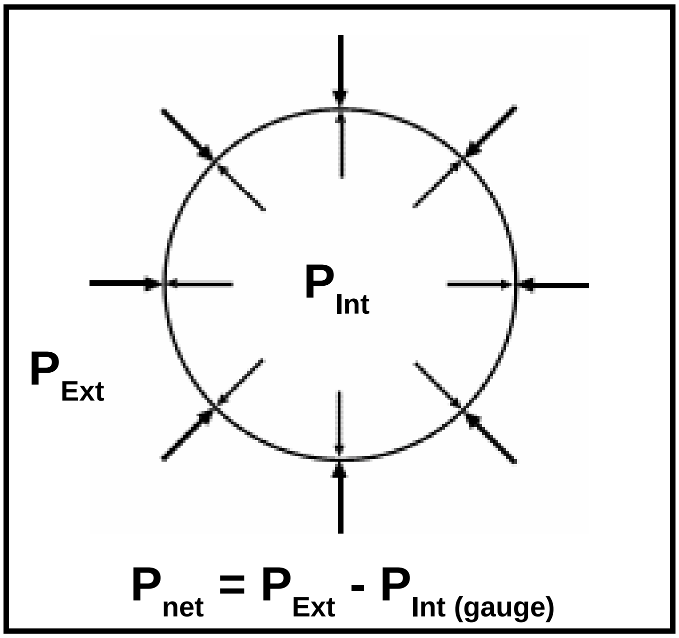
The first step in selecting the DR is to establish the load applied to the pipe. The next step is to select a trial DR and determine the pipe's resistance to the load. If the resistance has sufficient safety factor, the trial DR is acceptable. If not, calculations should be repeated with a lower DR until the safety factor is sufficient.

Applied Loads

Net External Pressure

The net external pressure, P_{net}, is the differential pressure between the inside and the outside of the pipe. Often external pressure applied by groundwater and soil is offset by fluid pressure within the pipe. Occasionally the net external pressure is greatly increased when a vacuum event occurs within the pipe. The designer must consider all possible combinations of internal fluid pressure, including transients, and external soil and groundwater pressure that may occur during the life and construction of the pipe and establish design net external pressures for each significant combination. Each of these design external pressures will be checked against the proposed DR to determine if an adequate safety factor exists. Each check will consider the load duration and whether loading is long-term, short-term, or between. Load duration has a significant affect on the pipe's response and strength.

Figure 2: Net External Pressure



Bore Hole Deformation

The earth pressure transmitted to the pipe will depend on the deformation of the bore hole. Research is still needed to define the parameters related to bore hole deformation, including factors affecting the stiffness of the residual mud-cuttings slurry and resultant earth loads. In soil or rock formations where little deformation occurs in the bore hole over the life of the pipeline, little or no earth pressure will be applied to the pipe (with the exception of the slurry head.) In many cases, load reaches the pipe as soil deforms inward into the bore hole. This process may eventually form an arch over the pipe which limits the load as in a tunnel. In other cases, arching may not occur. For instance, in shallow bore holes, surface pressure and vibration from live loads may ultimately collapse the bore hole. Likewise, a fluctuating groundwater level may cause a collapse. In some cases, the residual mud-cuttings slurry may resist large deformation of the bore hole. Determining the load applied to a directional drilled pipe requires considerable engineering judgment regarding the insitu soil formation, the bore hole fluids, and the mechanics of soil deformation. Part of the purpose of the geotechnical investigation should be to determine the anticipated earth pressures on the pipe after installation.

Earth Pressure

Earth pressure is applied to the pipe as the soil above the bore hole deforms into the borehole. Soil deformation usually results in a state of arching in the soil above the bore hole and the load applied to the pipe is significantly less than the prism load (geostatic pressure.) Generally the depth of cover is sufficient to develop arching where the pipe is at least five (5) pipe diameters deep, dynamic loads such as traffic or rail loads are absent or insignificant, and the soil has sufficient internal friction to develop arching. At the time of this writing, the HDD Industry has not developed exact equations for calculating earth pressure on HDD pipes. However, equations have been developed for jack-and-bore pipe that may be fairly applicable to HDD pipes. These equations may not always be conservative as there are differences between jack-and-bore and HDD. The designer is advised to give consideration to these differences and to determine for his/her own self their suitability. Stein [5] has published the following equation for estimating the earth pressure on jack-and-bored pipe (Terzaghi's equations are given along side Stein's as some HDD Designers use them.):

$$P_E = \frac{\kappa \gamma H}{144 \dfrac{inches^2}{ft^2}} \tag{1}$$

$$\kappa = \frac{1-\exp\left(-2\dfrac{KH}{B}\tan\delta\right)}{2\dfrac{KH}{B}\tan\delta} \tag{2}$$

where

P_E = external earth pressure, psi
γ = soil weight, pcf
H = depth of cover, ft.
κ = arching factor (Stein suggests $\kappa = 1$ for waterway crossings, i.e., the prism load)
B = borehole diameter, ft.
δ = angle of wall friction, degrees (For jack and bore pipe, Stein assumes $\delta = \phi/2$: Terzaghi assumed $\delta = \phi$ for buried pipe.)
ϕ = angle of internal friction, degrees
K = earth pressure coefficient. Stein suggests that $K = 0.5$ due to the minimal ground disturbance during directional drilling. Whereas, Terzaghi gives the following equation:

$$K = \tan^2\left(45 - \frac{\phi}{2}\right) \tag{3}$$

(If the effective soil weight is used the groundwater pressure must be added back in to get the total external pressure acting on the pipe. The effective soil weight is the dry unit weight of the soil for soil above the groundwater level; it is the saturated unit weight less the weight of water for soil below the groundwater level.)

Another approach to calculating earth pressure for jack-and-bored pipe is given in O'Rourke et al. [6]

Where arching does not occur, where the depth of cover is less than 5 diameters, or where liveloads are present the earth pressure may be estimated using the prism load per Eq. 7-1 in PLEXCO's Engineering Manual 2 (EM2). Liveload pressure may be calculated as given in Chapter 7 of EM2.

Hydrostatic Pressure due to Mud, Slurry, and/or Groundwater

In addition to earth pressure acting on the pipe, the mud, slurry, and/or groundwater that fills the annular space of the bore (that is, the space between the pipe OD and the bore hole ID) exerts an external hydrostatic pressure on the pipe. The pressure is equal to the fluid density times the height of the fluid head, which is the difference between the lowest elevation of the bore path curve and the elevation of the entry or exit pit, whichever is higher. (In some cases, the hydrostatic pressure from the mud-cuttings slurry may decrease if the slurry develops shear strength with time due to its thixotropy.)

Total External Pressure

The total external pressure acting on the pipe is the combination of the hydrostatic (mud, slurry and/or groundwater) pressure and the effective earth pressure. The mud pressure and earth pressure are typically calculated separately since there is no reduction in mud pressure due to arching.

HDD Pipe’s Resistance to Subsurface Loads

Resistance to Net External Pressure

Uniform external pressure acting on the pipe, as shown in Fig. 2, creates a ring compressive thrust that acts around the circumference of the pipe. In effect, the external pressure squeezes the pipe which causes compressive stress in the pipe wall. If this stress exceeds a critical threshold, the pipe can collapse. The collapse strength is inversely proportional to the cube of (DR-1). The designer must insure that the selected DR has a sufficient safety factor against collapse to withstand the net external pressure. However, the collapse strength is reduced by ring ovality in the pipe. Ovality occurs in coiled pipe and also when the pipe is subjected to non-uniform external pressure such as buoyant uplift forces, shipping and handling (parallel plate loads), and earth loads. Before the designer can check the collapse strength, the ovality must be determined.

Ovality or ring deflection generally results in a decrease in the pipe’s vertical diameter and an increase in its horizontal diameter. When buried, the increase in horizontal diameter is opposed by the passive resistance of the soil along side the pipe. The amount of deflection (ovality) depends on the stiffness of the soil and of the pipe. In the case of HDD, the soil is the residual mud-cuttings slurry in the borehole. While this mixture is thixotropic and will increase in viscosity if allowed to set undisturbed, determining its long-term stiffness is difficult. Until further research is done, it is conservative to assume that the stiffness modulus of the residual mud-cuttings slurry is similar to very soft clay. Soft clay lacks the stiffness to provide significant side-support for flexible pipe. Therefore, directional drilled pipe is assumed to have no support from the annular space mixture. The formulas given in the following sections are for calculating deflection and collapse resistance for pipe without side-support.

Deflection, collapse, and pulling strength are dependent on the material properties of PE particularly the modulus of elasticity and the tensile strength. Both of these properties are time-dependent. PE pipe’s resistance to a newly applied load increment decreases with time as the molecular structure rearranges due to viscoelasticity. This results in a higher resistance to short-term loading than to long-term loading. Careful consideration must be given to the duration and frequency of each load, so that the performance limit associated with that load can be calculated using PE material properties representative of that time period. For instance, during pull-back, the pipe’s tensile yield strength decreases with pulling time, so the safe (allowable) tensile stress is a function of time. See Table 1. (Typical values of the modulus of elasticity of HDPE for a wider range of temperature and the time are given in Table 5-1 of EM2.)

Table 1: Apparent Modulus of Elasticity and Allowable Tensile Stress @ 73^0F

Typical Apparent Modulus of Elasticity			Typical Allowable Tensile Stress		
Duration	HDPE	MDPE	Duration	HDPE	MDPE
Short-term	110,000 psi	87,000 psi	30 min	1300 psi	1000 psi
10 hours	57,500 psi	43,500 psi	60 min	1200 psi	900 psi
100 hours	51,200 psi	36,200 psi	12 hours	1150 psi	850 psi
50 years	28,200 psi	21,700 psi	24 hours	1100 psi	800 psi

Ring Bending Deflection

The most common equation used for calculating deflection of buried pipes is Spangler's Iowa formula. However, the Iowa formula was not developed with HDD applications in mind. It is based on a soil pressure distribution for pipe installed in earthen embedments and covered with backfilled soil, unlike HDD pipes. In addition, the parameters used with the Iowa formula, such as the soil modulus, lag factor, and bedding coefficient were empirically derived from embedded and backfilled pipe. They are generally not suitable for HDD pipes. HDD pipes typically are subjected to less earth pressure than backfilled pipes for the same depth of cover but, depending on the insitu soil and drilling muds, HDD pipes may have much less soil support both in the sense of soil uniformity and soil stiffness than embedded pipes.

In the following, it is assumed that (1) the earth load is applied at the pipe crown with a reaction at the invert and (2) the residual mud-cuttings slurry provides essentially no side-support. These assumptions imply that there is little horizontal pressure at the springline to resist vertical deflection which results from vertical load. Resistance to deflection is provided solely by the pipe's stiffness. There is likely some "cradling" of the pipe invert as the soil above the pipe pushes it into the bottom of the borehole. As soil deforms into the bore hole, soil pressure develops over a good portion of the pipe crown. Research is needed to develop an equation that will accurately describe the pipe's reaction to this load and determine how much, if any, support is provided by the slurry. Until this is done, the deflection will have to be approximated. Watkins and Anderson [7] give two ring deflection formulas for uniform loading on the top half of a pipe. One formula assumes the pipe's invert is placed on a rigid, flat base while the other assumes the invert reaction load is uniform around the bottom half of the pipe. For directional drilled pipe the load orientation is reversed, ie. the invert half of the pipe is likely to be uniformly loaded whereas the crown will have a more concentrated load, however the algebraic formula would be the same. The average of the two formulas gives the following approximation:

$$\frac{\Delta}{D} = \frac{0.0125P_E}{\frac{E}{12\ (DR-1)^3}} \qquad (4)$$

where

D = pipe diameter, in
Δ = Ring deformation, in
P_E = Earth pressure, psi
DR = Pipe Dimension Ratio
E = modulus of elasticity (typically, long-term), psi

Buoyant Deflection

Deflection may also occur in HDD pipe even when no earth load is present—for instance in some stiff-cohesive or lithified soils where the bore hole remains stable. An external pressure difference occurs between crown and invert when pipe is submerged in mud, slurry or groundwater (Archimedes's Principle). The pressure difference applies a force which deflects the invert upward toward the crown, thus creating ovality. Deflection is given by Eq. 5. This can be converted to percent deflection by multiplying it by 100. In some cases, buoyant deflection is additive with earth pressure deflection. Particularly, where buoyant deflection occurs during installation and hole collapse occurs over time. Normally, buoyant deflection can be virtually eliminated by filling the pipe with water.

$$\frac{\Delta}{D} = \frac{0.088\,\gamma_W D\,(DR\text{-}1)^4}{E\,(DR)} \tag{5}$$

where

Δ = ring deflection, in
D = pipe diameter, in
γ_W = weight of fluid in borehole, lbs/in^3
E = modulus of elasticity (typically, long-term), psi
DR = Dimension Ratio
(To convert fluid weight from lbs/ft^3 to lbs/in^3 divide by 1728.)

Reissner Effect

Longitudinal (beam) bending of a pipe induces ovality. For entrenched pipes, curvature is normally in the horizontal plane and the induced ovality is usually ignored as it is oriented transverse to earth load deflection. In a directional drilled pipe, curvature along the bore path induces ovality which is additive to earth load deflection. For DR 17 or heavier pipes, when the bending radius is greater than or equal to 40 pipe diameters, ovality is negligible and can be ignored.

Deflection Limits

Diametrical deflection is limited by geometric stability and by the bending strain induced in the pipe wall due to deflection. Geometric stability (collapse) is covered in the next section. L.E. Janson [8] observed that for pipes made with pressure-rated PE resins and subjected to soil pressure only, "no upper limit from a practical design point of view seems to exist for the bending strain". Therefore, for non-pressure pipes or conduits the only deflection limitation seems to be geometric stability. The maximum deflection a pipe can undergo before becoming unstable depends on a number of factors but typically instability occurs above 20% deflection in ground above the water table and above 15% deflection in ground below the water table. Normally, a safety factor is applied to stability. For instance, ASTM F-894, the standard for gravity-flow (non-pressure) profile wall HDPE pipes, gives a long-term deflection limit of 7.5%.

In addition to bending strains, pressurized pipes are subject to additional strains that occur when pressurization tends to reround the pipe. Due to these combined strains, it is conservative to limit pressurized pipe to less deflection than unpressurized pipe.

Design deflections are for use in selecting DR and for field quality control. (Field measured deflections exceeding the design deflection do not necessarily indicate unstable or over-strained pipe. In this case, the project engineer would have to analyze pipes to determine acceptability.)

Table 2: Design Deflection Limits of Buried Polyethylene Pipe, Long Term, %*

DR or SDR	21	17	15.5	13.5	11	9	7.3
Deflection Limit (%Dia) Non-pressure applications	7.5	7.5	7.5	7.5	7.5	7.5	7.5
Deflection Limit (%Dia) Pressure applications	7.5	6.0	6.0	6.0	5.0	4.0	3.0

*Deflection limits for pressure applications are equal to 1.5 times the short-term deflection limits given in Table X2.1 of ASTM F-714.

Unconstrained Collapse Resistance

The application of a uniform external pressure to the pipe as shown in Fig. 2 creates a ring compressive hoop stress in the pipe's wall. If the hoop stress exceeds a critical value, geometric instability occurs leading to reversal of curvature of the pipe wall. Constraining the pipe by embedding it in soil or cementitious grout will increase the pipe's collapse resistance. However, this Note assumes that the residual mud-cuttings slurry is generally not sufficiently stiff to provide such support. The following equation, known as Levy's equation, may be used to determine the allowable net external pressure (or negative internal pressure) for a directionally drilled pipe pulled into a borehole filled with slurry or non-cementitious grout:

$$P_{ua} = \frac{2E}{(1 - \mu^2)} \left(\frac{1}{DR - 1}\right)^3 \frac{f_O}{N} \qquad (6)$$

Where:

P_{ua} = allowable net external collapse pressure, psi

E = apparent modulus for the grade of material used to manufacture the pipe and for the time and temperature of the application, psi.

μ = Poisson's Ratio (Long term loading = 0.45, Short term loading = 0.35)

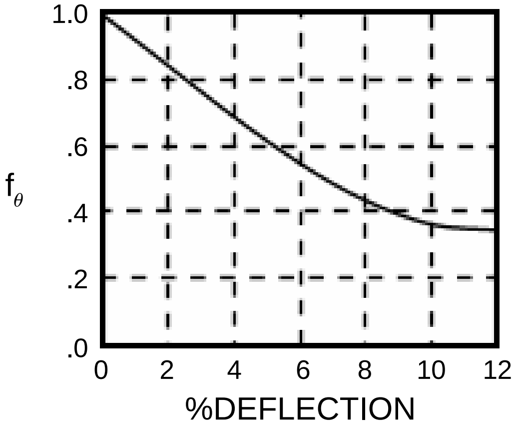
DR = Dimension Ratio (OD/t)

f_o = Ovality Compensation Factor (See below.)

N = safety factor, generally 2.0 or higher

Pipe ovality (deflection) reduces the pipe's resistance to external collapse pressure. Earth loads, longitudinal bending (bore path curvature), and buoyancy forces will produce ring deflection (ovality) in the pipe. Formulas for calculating earth load deflection and buoyancy deflection have been given in preceding sections. The Ovality Compensation Factor is given in Fig.3.

Figure 3: Ovality Compenstion Factor



This equation is suitable for use both during pullback and afterwards, however, during pullback there is a further reduction factor due to the pulling force in the pipe. This is discussed in a later section. If cementitious grout is used for the slurry, care must be taken to properly design and carefully execute the grouting process.

For design, the allowable collapse pressure, P_{ua}, must equal or exceed the net effective pressure, $P_{net.}$ If the safety factor in Levy's equation is set equal to one, the equation gives the critical collapse (buckling) pressure. Table 3 gives the critical collapse pressure for different DR's of HDPE pipe. For design purposes, the critical collapse pressure must be reduced by a safety factor and by ovality compensation to obtain an allowable stress, P_{ua}. When using Table 3 for determining pipe's resistance to buckling during pull-back, an additional reduction for tensile stresses is required, as given in the subsequent section, Reduced Collapse Strength during Pullback. Filling the pipe with water during the placement operation greatly reduces the possibility of pipe collapse during pullback.

As can be seen in Table 3 each DR shift (one DR higher or one DR lower) changes the buckling resistance by a factor of two. Design considerations for risk vs. economic gain should reflect the significant difference in collapse resistance between DR's.

After determining the DR required to handle the earth and groundwater pressures on the pipe during its service life, the designer must check to determine if the selected DR will meet all of the installation requirements.

Table 3: Critical Buckling (Collapse) Pressure for Unconstrained HDPE Pipe* @73°F (Table does NOT include Ovality Compensation or Safety Factor.)

Service Life	Pipe DR					
	Units	**7.3**	**9**	**11**	**13.5**	**17**
Short-term	psi	1003	490	251	128	61
	ft H_2O	2314	1130	579	296	141
	in Hg	2041	997	510	261	125
100 hrs	psi	514	251	128	66	31
	ft H_2O	1185	579	296	152	72
	in Hg	1046	511	261	134	64
1000 hrs	psi	438	214	110	56	27
	ft H_2O	1012	494	253	130	62
	in Hg	892	436	223	114	54
50 yrs	psi	283	138	71	36	17
	ft H_2O	653	319	163	84	40
	in Hg	576	281	144	74	35

* Full Vacuum is 14.7 psi, 34 ft water, 30 in Hg.
* Multipliers for Temperature Rerating:

60°F (16°C)	73.4°F (23°C)	100°F (38°C)	120°F (49°C)
1.08	1.00	0.78	0.63

*Axial Tension during pull-back reduces collapse strength. See Page 25.

INSTALLATION CONSIDERATIONS FOR DR SELECTION

Probably the most frequently asked questions of pipe manufacturers are “what force is needed to pull the pipe through my bore?” and “how much tensile pulling force can I apply to your pipe?”. PLEXCO can usually answer the second question but the first is another story. The safe pull force is a pipe property related to the tensile strength, temperature, and duration of load. (Other factors may reduce pulling strength including curvature in the pipe and external buckling forces.) However, the force required to pull the pipe through the bore is dependent on many factors such as the bore diameter, the soil, the slurry mixture, and drilling and pullback techniques. These factors are often site and job specific and therefore the pullback force must be determined by experienced drillers, geotechnical engineers, or consulting engineers familiar with the relevant site conditions and the drilling and pullback procedures. This Technical Note gives equations which PLEXCO believes to be the best available at the time of this publication. They are approximate and based on “ideal” conditions including a smooth bore path with no rough surfaces, a perfect borehole that is round and free of loose debris such as sand and gravel particles that can be trapped between the pipe and the borehole, and control of mud (bentonite) flow so that cuttings are removed effectively and gel does not occur until after pullback. **They will likely result in a calculated pullback force lower than the actual pullback force and there may be cases where they simply do not apply.** The user of these equations (including the design engineer and driller) assume all risks associated with their use. For final designs, PLEXCO recommends that the designer consult an experienced driller or engineer in determining the pullback force.

Bore Path Planning

The alignment of the bore path has a lot to do with the forces acting on the pipe during pullback. The straighter the alignment the less force on the pipe. Some curvature is necessary to get the pipe under the crossing obstacle, but curvature in any direction should be minimized. Curvature causes bending stress in the drill rods and in the service pipe and it significantly increases the pullback force because of the additional friction generated when pulling the pipe through a bend.

The bore path is determined after selection of the proper site. Site selection is based on a number of considerations, not the least of which is the determination whether or not directional drilling is suitable in the site soil. Bore path selection requires considerable engineering judgment and is beyond the scope of this document. See Hair [9] and NASTT Manual [1]. Such a decision may be based on previous experience at the site or on a geotechnical investigation. Location of all existing utilities and subsurface structures is essential. Site topography is important in establishing the site layout which must include room for stringing pipe, if not coiled, and for disposal and containment of mud and water. For river crossings, the extent of scour, meandering, and flooding must be established to provide adequate cover over the bore. Depth is also controlled by the concern of “fracking out” which results in the spillage of drilling fluid. The extent of the site investigation will depend on the pipe size, depth, bore length, site soils, and ultimate scope of the project. Waterway, marsh, and rail crossings are usually permitted by the appropriate regulatory agencies. Of particular concern to the driller are the location of existing utility lines, the identification of the boundary between rock and soil or hard and soft layers, presence of cobbles or boulders or other anomalies such as tree stumps, old foundations, and fill debris.

During drilling, a minimum cover depth should be maintained to ensure that no drilling fluid breakout occurs. For mini-HDD, this depth is typically 3 feet or greater but depends on specific site conditions. Curvature must be minimized at the entry and exit pits. Entry angles are controlled by the design of the rig and launching system and are typically between 12° and 15°. Whereas, exit angles must accommodate the pipe and are typically kept below 10°. The bore path should lay in a vertical plane. Normally, the bore path arcs down from the entry point (the “entry arc”), then levels off in a straight section, and finally arcs back up to the exit (the “exit arc”). The arcs and the straight length, ie. the bore path, must be defined for the boring rig operator.

Recommended Bending Radius

In addition to site constraints, drill path curvature is limited by the steering capabilities of the boring equipment, the bending radius of the drill rod, and the bending radius of the PE service pipe. Drilling rod typically has a recommended bend radius of $1200xD_{ROD}$, where D_{ROD} is the nominal rod diameter. The safe minimum bending radius of PE pipe is generally higher during pullback than for above-ground or open-cut installations. The Gas Research Institute recommends a minimum bending radius of $100xD_{PIPE}$ for PE pipe. When the bending radius is around $150xD_{PIPE}$ or less, the safe pulling strength of the pipe may be significantly reduced by the additional tensile stresses due to the curvature. See Axial Tensile Stress section, page 22.

Bore Path Geometry

The following equations may be of some use to the design engineer when laying out the bore path. However, these equations are approximations and the engineer is advised to use judgment when working with these equations.

The radius of curvature at any point along the bore path may be estimated by:

$$r = \frac{\Delta S}{\Delta \phi} \tag{7}$$

where

r = local radius of curvature along path, ft
ΔS = distance along path (pipe segment length), ft
$\Delta \phi$ = angular change in direction, radians.

The curvature of the entry and exit arcs may be estimated from the entry or exit angle and the depth of bore, as follows:

$$r_{avg} = \frac{2H}{\theta^2} \tag{8}$$

where

r_{avg} = average radius of curvature for entry or exit arc, ft
θ = bore entry or exit angle to surface, radians
H = depth of bore beneath surface, ft.

The corresponding horizontal distance required to achieve the depth may be estimated by:

$$L = \frac{2H}{\theta} \qquad (9)$$

where

L = horizontal transition distance, ft.

For small entry and exit angles (such as those mentioned as typical above), the bore path length can be approximated by L.

Pullback Consideration for DR Selection

During pullback large drill rigs can exert over 500,000 lbs of pulling force. Not all of this force is applied to the pipe. A good portion is necessary to pull the backreamer and displace or shear the soil. The amount of force applied to the pipe can be determined by placing a load cell between the swivel and the pipe. The pulling force on the pipe must overcome (1) frictional resistance between the pipe and the borehole (Fig. 4), (2) frictional resistance between the pipe and the ground surface, (3) "capstan effect" force, which is the result of increased bearing pressure of pipe being pulled around the inside curve of a bend (Fig. 5), (4) hydrokinetic drag, and (5) resistance due to pipe stiffness. The installer must estimate this force in order to size the drill rig and to set a limit on the maximum pulling force so that the pipe is not damaged during pullback. During the design phase, the designer may make the same estimate to select the optimum bore path and the proper pipe DR. In addition to the pulling force, longitudinal (beam) bending of the pipe along a curved bore path causes axial tensile stress due to bending. This stress adds to the pulling stress. During pullback, the pipe may be subjected to external pressures due to the fluid head of the mud and cutting mixture or groundwater. The pipe's collapse resistance given in Equation 12 is reduced by the axial pulling force due to the Poisson effect. Predicting the pullback force requires considerable judgment and is beyond the scope of these calculations. **The formulas presented in this Technical Note are based on the aforementioned "idealized" borehole and bore path and are for guidelines only. Pullback values obtained should be considered as qualitative values and used only for preliminary estimates. The calculated pullback value will likely be lower than the actual force required to achieve pullback. The designer and installer are advised to consult with an experienced driller or with an engineer familiar with calculating pullback forces.** Additional sources for information include programs such as DRILLPATH [10].

Figure 4: Frictional Resistance to Pullback

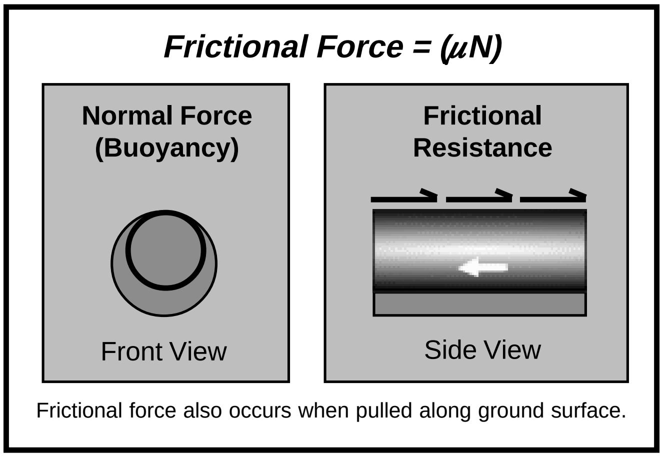
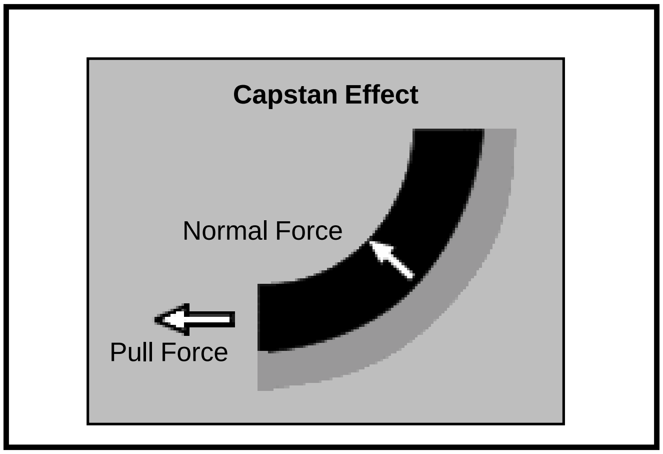


Figure 5: Capstan Effect



Pullback Force

The pullback force is normally calculated at the leading end of the pipe (i.e. just behind the pulling head). This force will increase as the pipe is pulled further into the bore. Although the tensile force may reach its highest value at the exit pit, the maximum tensile stress may occur elsewhere if there are additional tensile stresses due to bending of the pipe (curvature of the bore path). Equation 10 gives the frictional resistance or required pulling force, F_P, for pipe pulled in straight, level bores (no horizontal or vertical curvature) or across level ground. Typically, pull-back force calculations, including those given below, are approximations.

$$F_P = \mu w_B L \qquad (10)$$

Where

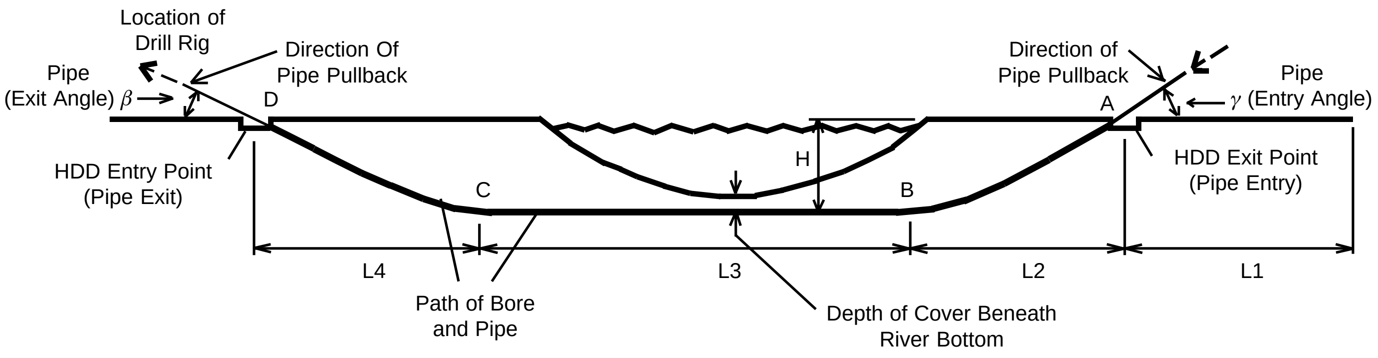
μ = coefficient of friction between pipe and slurry or between pipe and ground
w_B = net downward (or upward) force on pipe, lb/ft
L = length, ft

For pipe pulled around a curve or bend creating an angle θ, the capstan effect, F_C, can be accounted for using Equation 11.

$$F_C = e^{\mu\theta}(\mu w_B L) \qquad (11)$$

Most bores consist of a combination of straight sections and bends. Therefore it is necessary to apply Eq. 10 and Eq. 11 recursively to the pipe at each bend and section along the bore path. For a bore with no horizontal direction changes and the basic path as shown in Figure 6, there are four points A, B, C, D at which the equations are applied. The greatest force on the pipe would typically be at the pipe exit point D.

FIGURE 6: BORE PATH



The following method for estimating the loads for a bore such as that shown in Figure 6 with no horizontal bends has been presented by Larry Slavin of BellCore (Middletown, NJ): **(The user must determine the suitability of these equations for any specific application.)**

$$T_D = exp(v_b\beta)\ (T_C+T_{HK}+v_b|w_b|L_4-w_bH-exp(v_b\alpha)(v_aw_aL_4exp(v_a\alpha))) \qquad (12)$$

$$T_A = exp(v_a\alpha)\ (v_aw_a(L_1+L_2+L_3+L_4)) \qquad (13)$$

$$T_B = exp(v_b\alpha)\ (T_A+T_{HK}+v_b\,|\,w_b\,|\,L_2+\,w_b\,H\,-\,v_a\,w_a\,L_2\,exp\ (v_a\alpha)) \qquad (14)$$

$$T_C = T_B+T_{HK}+v_b|w_b|\ L_3-\ exp(v_a\alpha)(v_a\ w_a\ L_3\ exp(v_a\alpha)) \qquad (15)$$

where

T_A = pull force on pipe at point A, lbf
T_B = pull force on pipe at point B, lbf
T_C = pull force on pipe at point C, lbf
T_D = pull force on pipe at point D, lbf
T_{HK} = Hydrokinetic force, lbf. See below for hydrokinetic force equation.
L_1 = additional length of pipe required for handling and thermal contraction, ft
L_2 = horizontal distance to achieve desired depth, ft
L_3 = additional distance traversed at desired depth, ft
L_4 = horizontal distance to rise to surface, ft
H = depth of borehole from ground surface, ft
$exp(X) = e^X$, where e = natural logarithm base (e = 2.71828)
v_a = coefficient of friction applicable at the surface before the pipe enters borehole,
v_b = coefficient of friction applicable within the lubricated borehole or after the (wet) pipe exits,
w_a = weight of empty pipe, lbf/ft
w_b = net upward buoyant force on pipe in bore hole, lbf/ft
α = borehole angle at pipe entry (drill exit angle), radians, and
β = borehole angle at pipe exit (drill entry angle), radians.

If additional pipe length is pulled through the borehole to compensate for pipe contractions, pulling should be along the pipe exit angle. If the additional pull is not at the exit angle, the resulting total force would equal T_D, as calculated above, multiplied by exp($v_b\beta$) and the bearing pressure at the inside of the bend may cause pipe collapse.

The above equations do not explicitly account for the resistance due to the pipe stiffness at curves along the bore path. This effect will be reduced for sufficiently large radii and greater clearance within the bore hole, but may still represent a significant contribution.

Coefficient of Friction

A typical value for the coefficient of friction between PE pipe and a wet bore hole, v_b, is 0.3 and between PE pipe and ground, v_a, is 0.5. For pipe on rollers, this value may be reduced. See PPI Handbook Chapter 11. (Note: "typical values" are not exact determinations and have use only in estimating. The exact value should be determined by the designer or installer for each application.) The coefficient of friction also depends on whether or not the pipe is moving. The friction is the highest just prior to movement and decreases during movement. When pullback ceases, frictional forces and drag forces increase due to the thixotropic nature of the drilling mud, that is the mud's viscosity will increase when left undisturbed. An increase in viscosity could result in the pipe becoming stuck. Therefore, stoppage during pullback other than to change drill rods is discouraged.

The pullback force will depend upon whether the pipe is empty or deliberately weighted (e.g., filled with ballast) to reduce the buoyancy. Buoyant force pushes the pipe up against the top of the borehole and thus creates frictional drag between the pipe and the borehole. Filling the pipe with water reduces this force. The empty weight of the pipe may be obtained from the pipe manufacturer. The buoyant weight of the pipe may be calculated as given:

(Pipe Empty)

$$w_b = \frac{\pi D^2}{4}\gamma_b - w_a \tag{16}$$

where:

D = Pipe outer diameter, ft

γ_b = specific weight (lbf/ft^3) of the mud slurry and w_a equals the weight of empty pipe.

(Pipe full of fluid with unit weight equal to γ_f.)

$$w_b = \frac{\pi D^2}{4}\gamma_b - \frac{\pi D_I^2}{4}\gamma_f - w_a \tag{17}$$

where:

D_I = Pipe inside diameter, ft

To emphasize the importance of filling the pipe with water, an example solution is given in Table 4 which compares the calculated pullback force for a 24" DR 11 pipe empty and full of water using the BellCore equations for a hypothetical case of a 870 ft bore, 35 ft deep with 10^{o} entry and 15^{o} exit angles. Water is added as it is pulled below grade. For this example, the coefficient of friction on the surface was assumed to be 0.4 and in the bore hole assumed to be 0.25. Points A, B, C, and D refer to Figure 1. Table 4 shows that for this example the pullback force at the pipe exit hole almost doubles when the pipe is pulled empty.

Table 4: Theoretical Pullback Force for 24" DR 11 (870 ft Bore)

Point	Pullback Force Pipe Empty (lbf)	Pullback Force Pipe Filled with Water (lbf)
A	25,600	25,600
B	51,500	33,000
C	60,500	35,300
D	67,600	36,300

Hydrokinetic Pressure

During pullback, the pipe movement is resisted by the drag of the drilling fluid. The force is referred to as the hydrokinetic force and it is difficult to estimate. Hydrokinetic force depends on the drilling slurry, the slurry flow rate, and the dimensions of the pipe and the borehole. Typically, the hydrokinetic pressure is in the range of 5 to 10 psi. One equation proposed for calculating the hydrokinetic pressure follows:

$$T_{HK} = q \frac{\pi}{8} (D^2_{BH} - D^2) \qquad (18)$$

where

T_{HK} = hydrokinetic force, lbs
q = hydrokinetic pressure, psi
D_{BH} = borehole diameter, in
D = pipe outside diameter, in

Axial Tensile Stress

The maximum outer-fiber, tensile stress during pullback is obtained by taking the sum of the tensile stress due to the pullback force (which includes the hydrokinetic force) and the tensile stress due to bending at curves in the pipeline. The maximum tensile stress may not necessarily occur at the point of maximum pullback force. Where the bore path contains tight bends, a significant stress may occur in the pipe wall due to the curvature of the pipe. In this case, the allowable pullback force for the pipe may be considerably lower than the pullback force permitted in a straight run of pipe. The following equation may be used to determine the maximum tensile stress occurring in the pipe wall as a result of the combined affect of the pullback force and curvature:

$$\sigma_{PB} = \frac{T_i DR^2}{\pi D^2 (DR-1)} + \frac{ED}{2r} \qquad (19)$$

where

σ_{PB} = outer-fiber tensile stress, psi
T_i = pullback force at ith point, where i = A,B,C, or D
D = outer diameter of pipe, in
DR = Dimension Ratio
E = time-dependent modulus of elasticity, psi (See Table 2.)
r = radius of curvature of pipe along bore path, in.

The outer-fiber tensile stress, σ_{PB}, should not exceed the allowable tensile stress for the corresponding time period as given in Table 1. The term “safe pull strength” can be misleading, as the term implies that the allowable pulling force is a pipe property independent of the bore path and this is not the case. Bends in the bore path decrease the “safe pull strength” as they induce tensile stress in the pipe wall. This is illustrated in Table 5. Note the use of the term “safe pull force” rather than “safe pull strength”. The safe pull stress may be calculated by subtracting the bending stress due to curvature from the allowable tensile stress.

$$\sigma_{SP} = \sigma_{allow} - \frac{ED}{2r} \qquad (20)$$

Where
σ_{SP} = safe pull stress, psi
σ_{allow} = allowable tensile stress, psi. See Table 1.

The “safe pull force” can be found by multiplying the safe pull stress by the cross-sectional area of the pipe or as follows:

$$FS = \pi\sigma_{SP}\, D^2\left(\frac{1}{DR} - \frac{1}{DR^2}\right) \qquad (21)$$

Table 5: Radius of Curvature vs. Safe Pull Force for 8" DR 11 at σ_{allow} = 1200 psi.

Radius of Curvature (ft)	Bending Stress (psi)	Safe Pull Stress (psi)	Safe Pull Force (lbf)
50	400	800	15,450
100	200	1000	19,310
200	100	1100	21,250
400	50	1150	22,210
800	25	1175	22,690

The tensile strength of PE pipe is load-rate sensitive and therefore values of “safe” pull loads which might be satisfactory for sliplining or insertion renewal where the pull load is imposed for a maximum of 30 to 60 minutes may not be satisfactory for directional drilling. With directional drilling, the time duration of pulling stress application may be longer, between 4 hours to 24 hours. The “safe” pull-load is time dependent. Hence, the 60 min. or less “safe” pull load (to limit elongation in the forward portion of the pipeline where the pull force is largest), is inappropriate for longer duration pulls. The 24 hour value will normally keep the pull-nose “stretch” low and avoid localized herniation of the HDPE pipeline. Pullback values for gas pipe are given in ASTM F-1804, “Practice for Determining Allowable Tensile Load for Polyethylene (PE) Gas Pipe during Pull-In Installation”, but bending stress should be subtracted from the values in F-1804 using Eq. 20 to obtain the safe pullback stress.

After pullback, pipe may take several hours (typically equal to the duration of the pull) to recover from the axial strain. When pulled from the reamed borehole, the pull-nose should be pulled out about 3% longer than the total length of the pull. The elastic strain will recover immediately and the viscoelastic stretch will “remember” its original length and recover overnight. One does not want to come back in the morning to discover the pull-nose sucked back below the borehole exit level due to stretch recovery and thermal-contraction to an equilibrium temperature. In the worst case, the driller may want to pull out about 4% extra length (40 feet per 1000 feet) to insure the pull-nose remains extended beyond the borehole exit.

Break-away Links

The pulling forces at the drill rig will typically exceed the force acting on the pipe as force is required to pull the backreamer and drill-string. Generally, it is overly conservative to limit these forces to the pipe strength. Therefore, break-away links are placed between the swivel and the pipe to ensure that the allowable tensile loads are not exceeded. If the link breaks, it may be possible to remove the pipe from the entry end of the borehole. The break-away link must be rated so that is within the safe pulling load limit of the pipe; however, as described above, even with such a failsafe device present the pipe may be over stressed if it is pulled through too tight of a radius of curvature. The failsafe device will not detect over stressing due to bending.

Radially-directed External Pressure During Pullback

In addition to the tensile forces acting along the length of the pipe during pullback, the mud slurry head and the pumping pressure in the borehole (minus internal pressure in the pipe) create an external pressure acting on the pipe. Although this pressure results in a hoop compressive stress much lower than the tensile pulling stress, it is often significant. For safe design, during pullback the pipe's collapse resistance must exceed the net external pressure plus pumping pressure by a sufficient safety factor. The pipe's resistance to collapse has been discussed in a previous section. (The pumping pressure may vary. It is typically limited to about 10 psi.)

Unconstrained Collapse Resistance During Pullback

During pullback the collapse resistance of the pipe as given by Eq. 6 is reduced. The reduction occurs due to ovality and due to the hoop strain created by the pulling force, an affect due to Poisson's Ratio. Ovality is always significant but can be large when the pipe is pullback empty and deformed by buoyant pressure. Figure 1 gives the reduction in collapse resistance due to ovality. The reduction due to the tensile pulling force can be accounted for by an additional reduction factor, f_R. Multiply Eq. 6 (P_{ua}, allowable external collapse pressure) by the reduction factor, f_R to obtain the allowable external buckling pressure during pull-back.

$$f_R = \sqrt{(5.57-(r+1.09)^2)} - 1.09 \qquad (22)$$

$$r = \frac{\sigma_T}{2\sigma_{SP}} \qquad (23)$$

Where f_R = reduction factor caused by axial stress
σ_T = calculated tensile stress during pull-back, psi
σ_{SP} = safe pull stress, psi

Torsional Stress

During pullback and reaming, a swivel is typically used to separate the rotating cutting head assembly from the pipeline pull segment. Swivels are not 100% efficient and some minor percent of torsion will be transmitted to the pipeline. For thick wall HDPE pipes of SDR 17, 15.5, 11, 9 and 7 this torsion is not significant and usually does not merit a detailed engineering analysis. If one is made, a typical value for allowable torsional shear stress is 50% of the allowable tensile stress. The torsional shear stress intensity may be obtained by dividing the transmitted torque by the wall area.

Pulling Heads

A number of manufacturers make pulling heads that can be attached to PE pipe. Pulling heads should be designed so that the pullback force is uniformly transmitted to the pipe and stress concentrations on the surface of the pipe are minimized.

Pipe Inspection

After pullback, the installer will ordinarily pull extra length of the pipe out of the exit end of the borehole to allow for shrinkage due to relaxation, as described above. The pipe should be inspected for roundness (grossly ovaled or flattened pipe generally indicates over pulling and collapse), for surface scratches, and for necking. Deep scratches or gouges can reduce the pressure rating of the pipe or act as stress risers. The American Gas Association's Plastic Pipe Manual states that "a commonly used rule of thumb requires that defects greater than 10% of the wall thickness should not be installed." In some cases, this rule of thumb may be too restrictive and an engineering judgment need be made.

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APPENDIX 1 - EXAMPLE CALCULATIONS

(The following examples are for instructional purposes only. They represent hypothetical cases and are not design calculations. In real applications, the design engineer must determine the applicability of the equations given in this Technical Note and the desired level of safety factor based on the critical nature of the application, the nature of the soil formation, the accuracy and availability of geotechnical data, the experience record of the drilling contractor, and other factors.)

Example 1: An 8" IPS DR 11 Yellowstripe pipe is installed under a rail spur. The depth of bore is 12 ft. Estimate deflection and determine the safety factor against buckling. The following calculations assume no internal pressure inside the pipe. Internal pressure will cause rerounding but more significantly it reduces the net external pressure causing buckling.

Given Parameters:

OD =8.625 in	Nominal Pipe OD	DR = 11	Pipe Dimension Ratio
H = 12 ft	Max. Borehole Depth		
γ = 110 lbf/ft^3	Unit Weight of Soil	P_{live} = 800 lbf/ft^2	E-80 Live Load at 12 ft

PE Material Parameters:

Wheel loading from train will be applied for several minutes without relaxation. Repetitive train crossings may accumulate. For this example, assume the frequency of trains crossing justifies the use of the 100-hour modulus.

E_{mid} = 51200 psi μ = 0.45 Long Term Poisson's Ratio

Soil and Live Load Pressure on Pipe

Assume that because of the dynamic surface loading the borehole deforms applying the prism load and the live load. See Eq. 7-1 PLEXCO's EM2.

$$P_E = (\gamma H + P_{Live})\left(\frac{1\ \text{ft}^2}{144\ \text{in}^2}\right) \qquad P_E = 14.7\ \text{psi}$$

Ring Deformation resulting from soil and live load pressures is given by Eq. 4.

$$\%\Delta V: = \frac{0.0125\ P_E}{\left[\dfrac{E_{mid}}{12\ (DR - 1)^3}\right]}(100) \qquad \%\Delta V = 4.3$$

Percent vertical deflection is less than the allowable given in Table 2 for DR 11.

(A more exact calculation of deflection would include compensation for rerounding due to internal pressure. Such a calculation is beyond the scope of this Technical Note.)

Determine the safety factor against collapse using Eq. 6:

f_0=0.67 Ovality compensation factor for 4.3% ovality from Figure 3.

$$P_{UC} = \frac{2E_{mid}}{(1-\mu^2)}\left(\frac{1}{DR-1}\right)^3 f_0 \qquad P_{UC} = 86.0\ \text{psi}$$

Critical unconstrained buckling pressure (no safety factor)

$$SF_{cr} = \frac{P_{UC}}{P_E} \qquad SF_{cr} = 5.9$$

Safety factor against buckling

Example 2: A 8" IPS DR 13.5 Redstripe conduit pipe is being pulled under a river. The maximum depth of bore is 18 feet below the river surface. Bentonite slurry is used to keep the hole open. Its weight is 75 lb/cu.ft. The duct is empty during the pull. Calculate a) the safe pull force and b) the safety factor against buckling during pullback for the pipe. Assume that the pipe's ovality is 3% and that the pulling time will not exceed 10 hours. The borepath has a radius of curvature in excess of 1000 ft. so bending stresses may be ignored.

Solution:

a) Calculate the safe pull force.

$OD = 8.625$ in Pipe outside diameter $DR = 13.5$ Pipe dimension ratio

$\sigma_{SP} = 1150$ psi Typical safe pull stress for HDPE for 12-hour pull duration from Table 1.

$$FS = \pi \sigma_{SP} OD^2 \left(\frac{1}{DR} - \frac{1}{DR^2} \right)$$

Equation 21

$F_S = 18{,}434$ lbf Safe pull force for 8" IPS DR 13.5 HDPE pipe assuming 10-hour maximum pull duration

b) Step 1) Determine the critical buckling pressure during pullback of the pipe (include tensile reduction factor assuming the frictional drag during pull results in 1000 psi tensile stress in the pipe.

$E = 57500$ psi Apparent modulus of elasticity (for 10 hours at 73 degrees F)

$\mu = 0.45$ Poisson's ratio (long term value)

$f_0 = 0.76$ Ovality compensation factor (for 3% ovality)

$\sigma_T = 1000$ psi Assumed pull stress

$$r = \frac{\sigma_T}{2\,\sigma_{SP}}$$

$r = 0.43$ Tensile ratio coefficient Eq. 23

$$f_R = \sqrt{5.57 - (r + 1.09)^2} - 1.09$$

$f_R = 0.72$ Tensile reduction factor, Eq. 22

$$P_{cr} = \frac{2E}{(1 - \mu^2)} \left(\frac{1}{DR - 1} \right)^3 f_0 f_R$$

$P_{cr} = 40.4$ psi Critical unconstrained buckling pressure for DR 13.5 pipe Without safety factor from Eq. 6.

Step 2) Determine expected loads on pipe (assume only static drilling fluid head acting on pipe, and borehole intact with no soil loading during pullback.) ,

$\gamma_{Slurry} = 75$ lbf/ft^3 Drilling fluid weight $H = 18$ ft Maximum bore depth

$$P_{slurry} = H \gamma_{Slurry} \left(\frac{1 \text{ ft}^2}{144 \text{ in}^2} \right)$$

$P_{slurry} = 9.4$ psi Total static drilling fluid head pressure if drilled from surface

Step 3) Determine the resulting safety factor against critical buckling during installation

$$SF_{cr} = \frac{P_{cr}}{P_{slurry}}$$

$SF_{cr} = 4.3$ Safety factor against critical buckling during pull

Example 3 Determine the safety factor for long term performance for the Redstripe conduit in Example 2 for the section of pipe under the river. Assume there are 10 feet of river bed deposits above the borehole having a saturated unit weight of 110 lb/cu.ft (18 feet deep, 3% initial ovality) (Depending on the bore path, pipe in the river bank may have higher loads than pipe in the river.)

Solution:

Step 1) Determine the load on pipe due to saturated river sediments

$E_{long} = 28200$ psi	Long term apparent modulus	$\gamma_w = 62.4$ lbf/ft^3	Unit weight of water
$H = 18$ ft	Max borehole depth	$\gamma_{ss} = 110$ lbf/ft^3	Saturated unit weight of sediments
$GW = 18$ ft	Depth below river surface		
$H_C = 10$ ft	Height of soil cover	$OD = 8.625$ in	Nominal pipe OD
$DR = 13.5$	Pipe dimension ratio	$\mu = 0.45$	Long term Poisson's ratio

$$P_{soil} = (\gamma_{ss} - \gamma_W)\, H_C \left(\frac{1\text{ ft}^2}{144\text{ in}^2}\right)$$

$P_{soil} = 3.3$ psi Prism load on pipe from 10' of saturated cover (including buoyant force on submerged soil)

Step 2) Calculate the ring deflection. Use the larger of the deflection resulting from (a) soil loads assuming no side support or from (b) buoyant deflection due to mud weight.

$$\%\Delta V = \frac{0.0125\, P_{soil}}{\left[\dfrac{E_{long}}{12\,(DR-1)^3}\right]}(100)$$

$\%\Delta V = 3.4$ Ring deformation from Eq. 4

$$\%\Delta V_b = \frac{8.77\ \gamma_{ss}\ OD\ (DR-1)^4}{E_{long}\ DR}\left(\frac{1\text{ft}^3}{1728\text{ in}^3}\right)$$

$\%\Delta V_b = 0.31$ Buoyant deformation from Eq. 5

Step 3) Determine the long-term hydrostatic loads on the pipe

$$P_w = \frac{GW}{2.31\text{ ft/psi}} + P_{soil}$$

$P_w = 11.1$ psi External pressure due to water head and buoyant weight of river sediments.

$\gamma_{slurry} = 75$ lbf/ft^3 Unit weight of drilling fluid

$$P_{slurry} = \gamma_{slurry}\ H\left(\frac{1\ ft^2}{144\ in^2}\right)$$ $P_{slurry} = 9.4$ psi External pressure due to slurry head

$P_w > P_{slurry}$, therefore use P_W for buckling load

Step 4) Determine safety factor against buckling using Eq. 6

$f_0 = 0.72$ Ovality compensation from Fig. 3 based on ovality calculated in Step 2.

$$P_{UC} = \frac{2E_{long}}{(1-\mu^2)}\left(\frac{1}{DR-1}\right)^3 f_0$$ $P_{UC} = 26.1$ psi Critical unconstrained buckling pressure (no safety factor)

$$SF_{cr} = \frac{P_{UC}}{P_W}$$ $SF_{cr} = 2.4$ Safety factor against buckling pressure of highest load (slurry)

(8" pipe is normally not supplied in coils. Where coiled pipe is used, the pipe will retain some ovality from the coiling process unless it is mechanically rerounded. Ovality due to coiling is additive to ovality due to deflection.)

Example 4: A 24" IPS DR 11 Bluestripe pipe is being pulled through a 36" (3ft) bore hole drilled in medium-dense, clayey sand at 25 ft below the ground surface. The angle of internal friction for the soil formation is assumed to be 30^0 and the ground water is to the surface. Estimate deflection and the safety factor against collapse for the following cases: (1) Assume arching occurs. (2) Use the same assumption as case 1, while the pipe is under a full vacuum. (This would be the case when the pipe is subjected to negative pressure during a water hammer event) For both cases, assume there is no live load present.

Given Parameters:

OD =24 in	Nominal Pipe OD	γ = 130 lbf/ft^3	Saturated Unit Weight of Soil
H = 25 ft	Max Borehole Depth	γ_w = 62.4 lbf/ft^3	Unit Weight of Water
DR = 11	Pipe Dimension Ratio	H_w = 25ft	Height of Groundwater

PE Material Parameters:

Case 1: The long term elastic modulus will be used due to the constant soil pressure.

E_{long} = 28200 psi	Long Term Elastic Modulus
μ_{long} = 0.45	Long Term Poisson's Ratio

Case 2: Since water hammer events cause a temporary pressure loss within the pipe, the short-term elastic modulus will be used.

E_{short} = 110000 psi	Short Term Elastic Modulus
μ_{short} = 0.35	Short Term Poisson's Ratio

Estimated Soil Pressure on Pipe

Assume a depth of cover of at least five (5) pipe diameters deep is adequate to cause sufficient internal friction within the soil to develop arching. See EQ. 1 and 2.

Case 1:

ϕ = 30 deg	Internal Angle of Friction
B = 3ft	Diameter of Borehole (36")
$\delta = \phi/2$	Angle of Wall Friction (Stein suggests a value of $\delta=\phi/2$)
K = 0.5	Earth Pressure Coefficient (Stein suggests K = 0.5)

Approximate the arching factor using Eq. 2:

$$\kappa = \frac{1 - \exp\left(-2\frac{KH}{B}\tan\delta\right)}{2\frac{KH}{B}\tan\delta} \qquad \kappa = 0.4$$

Approximate the external earth pressure using Eq. 1:

$\gamma_{eff} = \gamma - \gamma_w$ $\gamma_{eff} = 67.6$ lbf/ft3 Effective Soil Weight

$$P_E = (\gamma_{eff} H \kappa)\left(\frac{1\ ft^2}{144\ in^2}\right) \qquad P_E = 4.7\ psi$$

Ring Deformation resulting from soil pressure is estimated by Eq. 4.

$$\%\Delta V = \frac{0.0125\ P_E}{\left[\frac{E_{long}}{12\ (DR - 1)^3}\right]}(100) \qquad \%\Delta V = 2.5$$

Percent vertical deflection is less than the allowable given in Table 2 for DR 11 pipe.

(A more exact calculation of deflection would include compensation for rerounding due to internal pressure. Such a calculation is beyond the scope of this Technical Note.)

Estimate the safety factor against collapse using Eq. 6:

$f_0 = 0.8$ Ovality compensation factor for 2.5% ovality from Figure 3

$$P_{UC} = \frac{2E_{long}}{(1-\mu_{long}^2)}\left(\frac{1}{DR-1}\right)^3 f_0 \qquad P_{UC} = 56.6psi$$

Critical unconstrained buckling pressure (no safety factor)

$$P_N = P_E + (\gamma_w \cdot H_w)\left(\frac{1\ ft^2}{144\ in^2}\right) \qquad P_N = 15.5$$

Net External Pressure

$$SF_{cr} = \frac{P_{UC}}{P_N} \qquad SF_{cr} = 3.7$$

Safety factor against buckling

CASE 2:

$\phi = 30$ deg	Internal Angle of Friction
B = 3ft	Diameter of Borehole (36”)
$\delta = \phi/2$	Angle of Wall Friction (Stein suggests a value of $\delta = \phi/2$)
K = 0.5	Earth Pressure Coefficient (Stein suggests K = 0.5)

Estimate the arching factor using Eq. 2:

$$\kappa = \frac{1 - \exp\left(-2 \frac{KH}{B} \tan\delta\right)}{2 \frac{KH}{B} \tan\delta} \qquad \kappa = 0.4$$

Estimate the external earth pressure using Eq. 1:

$\gamma_{eff} = \gamma - \gamma_w$ $\quad\gamma_{eff} = 67.6$ lbf/ft3 Effective Soil Weight

$$P_E = (\gamma_{eff} H \kappa)\left(\frac{1\ \text{ft}^2}{144\ \text{in}^2}\right) \qquad P_E = 4.7\ \text{psi}$$

Total Pressure

$P_{int} = 14.7$ psi Internal Pressure of Pipe (total vacuum)

$P_{net} = P_E - P_{int}$ $\quad P_{net} = 19.4$ psi

Estimate Ring Deformation resulting from soil pressure is estimated by Eq. 4. Vacuum does not contribute to ring deflection.

$$\%\Delta V = \frac{0.0125\ P_E}{\left[\frac{E_{long}}{12\ (DR - 1)^3}\right]}(100) \qquad \%\Delta V = 2.5$$

Percent vertical deflection is less than the allowable given in Table 2 for DR 11 pipe.

(A more exact calculation of deflection would include compensation for rerounding due to internal pressure. Such a calculation is beyond the scope of this Technical Note.)

Estimate the safety factor against collapse using Eq. 6:

$f_0 = 0.8$ Ovality compensation factor for 2.5% ovality from Figure 3

$$P_{UC} = \frac{2E_{short}}{(1-\mu_{short}^2)}\left(\frac{1}{DR-1}\right)^3 f_0 \qquad P_{UC} = 200.6\ \text{psi}$$

Critical unconstrained buckling pressure (no safety factor)

$$SF_{cr} = \frac{P_{UC}}{P_{net}} \qquad SF_{cr} = 10.3$$

Safety factor against buckling

Example 5: Using Slavin’s Method, find the estimated force required to pull-back 8” DR 11 HDPE pipe and the safety factor against collapse for the river crossing shown in figure 6. Assume the HDPE pipe is 20 It deep and approximately 500 ft long with a 8 deg. entry angle and a 12 deg. exit angle. The actual pull-back force will vary depending on the size, selection, and use of the backreamer, bore hole stability; soil conditions; betonnite lubrication; driller expertise; and other application circumstances.

Pipe Properties

Outside Diameter	$D = 8.625$ in	Long-term Modulus	$E_{long} = 28250$ psi
Standard Dimension Ratio:	$DR = 11$	24 hr Modulus	$E_{24hr} = 56500$ psi
Minimum wall thickness:	$t = 0.784$ in	Poisson’s ratio (long term)	$\mu = 0.48$
		Safe Pull Stress (24 hr)	$\sigma_{pb} = 1100$ psi

Path Profile:

$H = 20$ ft	Depth of bore
$\theta_{in} = 8$ deg	Pipe entry angle
$\theta_{ex} = 12$ deg	Pipe exit angle
$L_1 = 100$ ft	Pipe drag on surface (This value starts at total length of pull, approximately 500 ft then decreases with time. Assume 100 It remaining at end of pull)
$L_{cross} = 500$ ft	

Estimated Path length (Determine L2 and L4):

Estimated Average Radius of Curvature for Path at Pipe Entry (Eq. 8)

θ is given in radians

$$R_{avgin} = \frac{2H}{\theta_{in}^{2}} \qquad R_{avgin} = 2051.6 \text{ ft}$$

Estimated Average Radius of Curvature for Path at Pipe Exit (Eq. 8)

$$R_{avgex} = \frac{2H}{\theta_{ex}^{2}} \qquad R_{avgex} = 911.9 \text{ ft}$$

Estimated Horizontal Distance Required to Achieve Depth or Rise to the Surface at Pipe Entry (Eq. 9)

$$L_2 = \frac{2H}{\theta_{in}} \qquad L_2 = 286.5 \text{ ft}$$

Estimated Horizontal Distance Required to Achieve Depth or Rise to the Surface at Pipe Exit (Eq. 9)

$$L_4 = \frac{2\,H}{\theta_{ex}} \qquad\qquad L_4 = 191.0\ \text{ft}$$

where

L_2 & L_4 = horizontal transition distance at bore exit & entry respectively

Approximate Axial Bending Stress:

$R = R_{avgex}$ Min. Radius for Bore path.

$R = 911.9$ ft

$D = 8.625$ in

Radius of curvature should exceed 40 times the pipe outside diameter to minimize ring kinking.

$r = 40\ D$

$r = 28.8$ ft Okay. $R > r$

Bending strain

$$\varepsilon_a = \frac{D}{2\,R} \cdot \frac{(1\text{ft})}{(12\text{in})} \qquad\qquad \varepsilon_a = 3.9 \cdot 10^{-4}\ \text{in/in}$$

where

ε_a = bending strain, in/in

D = outside diameter of pipe, in

R = minimum radius of curvature, ft

Bending stress

$$\sigma_a = E_{24hr}\,\varepsilon_a \qquad\qquad \sigma_a = 22.0\ \text{psi}$$

where

σ_a = bending stress, psi

Estimate Pulling Force:

Weight of Empty Pipe

$\rho_w = 3.6 \cdot 10^{-2}\ \text{lbf/in}^3$

$\gamma_a = 0.95$ $\gamma_b = 1.5$

$$w_p = \pi\,D^2\,\frac{DR - 1}{(DR)^2}\,\rho_w\,\gamma_a\ (12\ \text{in/ft}) \qquad\qquad w_p = 7.9\ \text{lbf/ft}$$

Note: The average wall thickness equals 1.06 times the minimum wall.

$w_a = 1.06\ w_p$ $w_a = 8.4$ lbf/ft Average weight per foot

Net Upward Buoyant Force on Empty Pipe Surrounded by Mud Slurry (Eq. 16)

$$w_b = \pi \frac{D^2}{4} \rho_w \gamma_b \frac{(12in)}{(1ft\ )} - w_a \qquad w_b = 29.5 \text{ lbf/ft}$$

where

ρ_w = density of water, lbf/in^3

γ_a = specific gravity of the pipe material

γ_b = specific gravity of the mud slurry

w_a = weight of empty pipe, lbf/ft

W_b = net upward buoyant force on empty pipe surrounded by mud slurry

Estimate pullback force acting on pipe applying Eq. 12 through Eq. 15

See figure:

$L_1 = 100$ ft $\nu_a = 0.4$

$L_2 = 286.5$ ft $\nu_b = 0.25$

$L_3 = 200$ ft $\alpha = \theta_{in}$ $\alpha = 8$ deg

$L_4 = 191.0$ ft $\beta = \theta_{ex}$ $\beta = 12$ deg

$L_3 = L_{cross} - L_2 - L_4$ $L_3 = 22.5$ ft

Estimate Hydrokinetic Pressure per Eq. 18

$q = 10$ psi

$D_{BH} = 1.5\ D$ $D_{BH} = 12.9$ in

$$T_{HK} = q \frac{\pi}{8} (D_{BH}^{\ 2} - D^2) \qquad T_{HK} = 361.4 \text{ lbf}$$

Estimate force at Point A per Eq. 13

$T_A = \exp(\nu_a\ \alpha)\ [\nu_a\ w_a.\ (L_1 + L_2 + L_3 + L_4)\ ]$

$T_A = 2131.8$ lbf

Estimate force at Point B per Eq. 14

$T_B = \exp(\nu_b\ \alpha)\ (T_A + T_{HK} + \nu_b\ |\ w_b |\ L_2 + w_b\ H - \nu_a\ w_a\ L_2 \exp(\nu_a\ \alpha))$

$T_B = 4326.6$ lbf

Estimate force at Point C per Eq. 15

$T_C = T_B + T_{HK} + \nu_b \mid w_b \mid L_3 - \exp(\nu_b \alpha)\ (\nu_a\ w_a\ L_3 \exp(\nu_a \alpha))$

$T_C = 4771.2$ lbf

Estimate force at Point D per Eq. 12

$T_D = \exp(\nu_b \beta)\ [T_C + T_{HK} + \nu_b \mid w_b \mid L_4 - w_b H - \exp(\nu_b \alpha)\ (\nu_a\ w_a\ L_4 \exp(\nu_a \alpha))]$

$T_D = 5.530.6$ lbf

where

L_1 = pipe on surface, ft

L_2 = horizontal distance to achieve desired depth, ft

L_3 = additional distance traversed at desired depth, ft

L_4 = horizontal distance to rise to surface, ft

ν_a = coefficient of friction applicable at the surface before the pipe enters bore hole

ν_a = coefficient of friction applicable within the lubricated bore hole or after the (wet) pipe exits

α = bore hole angle at pipe entry, radians

β = bore hole angle at pipe exit, radians (refer to figure 1)

Compare Axial Tensile Stress due to Pullback Force with Allowable Tensile Stress (1100 psi):

Average Estimated Axial Stress Acting on Pipe Cross-section at Points A, B, C, and D per Eq. 19

$$\sigma_i = (T_i)\frac{1}{\pi D^2}\ \frac{(DR)^2}{DR - 1} + \frac{E_{24hr} D}{2R \cdot \frac{(12in)}{(1ft)}}$$

$\sigma_1 = 132.6$ psi <1100psi OK

$\sigma_2 = 256.3$ psi <1100psi OK

$\sigma_3 = 269.3$ psi <1100psi OK

$\sigma_4 = 308.6$ psi <1100 psi OK

where

$T_i = T_A, T_B, T_C, T_D$, lbf

σ_i = corresponding stress, psi

Breakaway links should be set so that pull-back force applied to pipe does not exceed 1100 psi stress.

$$ID = D - 2\,t$$

$$F_b = \sigma_{pb}\,\frac{\pi}{4}(D^2 - ID^2) \qquad F_b = 21{,}243 \text{ lbf}$$

Estimate safety factor against ring collapse during pull-back

External Hydraulic Load

External static head pressure

$$P_{ext} = \rho_w \cdot \gamma_b \cdot H \qquad P_{ext} = 13 \text{ psi}$$

Combine static head with hydrokinetic pressure to find maximum pressure during pullback

$$P_{max} = P_{ext} + q \qquad P_{max} = 23 \text{ psi}$$

Find the estimated critical collapse pressure using Eq. 6, Eq. 22, and Eq. 23.

Resistance to external hydraulic load during pull-back

$f_0 = 0.76$ Ovality compensation factor (for 3% ovality)

Tensile ratio (based on assumed 1100 psi pull stress calculation) (Eq. 23)

$$r = \frac{\sigma_4}{2\,\sigma_{pb}} \qquad r = 0.14$$

$$f_R = \sqrt{5.57 - (r + 1.09)^2} - 1.09 \qquad f_R = 0.92$$

Tensile reduction factor (Eq. 22)

Estimated collapse pressure with reduction for tensile pulling force.

$$P_{cr} = \frac{2\,E_{24hr}}{(1 - \mu^2)}\left(\frac{1}{DR - 1}\right)^3 f_0\,f_R \qquad P_{cr} = 102.7 \text{ psi}$$

Safety factor against collapse

$$SF = \frac{P_{cr}}{P_{max}} \qquad SF = 4.5$$